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Project No. 1455

Mr. Leo Suver
Burnstead Construction Company
11980 NE 24th St., Ste. 200
Bellevue, WA 98005

Subject: Geotechnical Report
Emrick Property
7303 Renton-Issaquah Road (Hwy. 900)
Parcel #20010619900001
Issaquah, Washington

Dear Mr. Suver:

As requested, we have conducted a geotechnical engineering study for the proposed residential construction on the Emrick property in Issaquah, Washington. Our original report was revised based on new site and grading plans developed in 2020. We understand that the project plans call for the development of twenty four residential lots and two detention vaults on the sloping ridge above Highway 900. This report presents our findings and recommendations for the geotechnical aspects of project design and construction of the residences on the property.

SUMMARY

Our field exploration indicates the site is generally underlain by medium dense silty sands followed by a layer of dense glacial till-like material along the ridge area where the development will occur. Some areas of looser silty soils underlie the lower eastern section of the development. Areas further uphill to the west include several wetlands, a creek and buffer area, and an extensive steep slope area in the far western portion of the site.

The western steep slope extends up to one of the Talus Housing Divisions. One area of the planned roadway of the development near the lower wetland is underlain by a stiff silt which may be a type of pond deposits from the Ice Contact Drift in the area. Only minor groundwater seepage was noted in one of the pits following a relatively dry period in the winter of 2017. The test pits extended to depths of up to 10.5 feet. The test borings extended to depths of about 56 feet beneath the surface. Groundwater was encountered in the borings at depths of 15 to 18 feet during the drilling at the end of February 2017.

In our opinion, the site conditions encountered are suitable for the planned residential construction on the site. Due to the presence of some regulated 40% slopes along the lower east margin of the site along Highway 900, appropriate slope setbacks and buffers should be provided for siting the residences. We understand that some stream/wetland buffers will be used in some of the higher areas on the site.

The undisturbed native soils are suitable for supporting the proposed residences, access driveways, structural walls and utilities provided the recommendations presented in this report are incorporated into project design and construction.

Generally, most of the onsite soils have a significant percentage of "fines" and will not be suitable for use as structural fill in wet weather conditions. Excavations at the site may be made with conventional excavating equipment. In addition, due to the high fines content, onsite soil conditions are not suitable for storm water infiltration.

The geotechnical recommendations presented in this report should be incorporated into project design and construction.

PROJECT DESCRIPTION

The currently proposed layout is shown on drawings (Sheets C4.01 and C4.02) prepared by CORE Design, dated September 16, 2020. The project layout shows twenty-four residential lots. These will be generally located along a ridge-like topographic feature, primarily in the northern portion of the site. The southwestern, eastern and southern parts of the property include wetlands and their buffers, a stream and its buffer, and the steep western slopes. A 35-foot right of way and roadway will access the lots and connect with the southeastern access easement up the slope from Highway 900 below.

Detailed building plans for the residences have not been prepared. Daylight lower level homes will be constructed on some of the lots due to ground surface gradients in the areas of home placement. These are specifically anticipated for Lots 21-24. We expect the new residences will have conventional spread footing foundations and concrete slab-on-grade garage floors. Some of the lower level building walls may be basement retaining walls.

We expect that new houses may be two-story wood frame construction and that the ground floor level will be established at or near the existing grade or on a daylight level. Building loads are expected to be light, on the order of one to two kips along building walls and 20 kips at columns.

Two detention vaults are planned along the alignment of the access roadway. The northern vault will have a plan area of 2000 square feet and a maximum depth of about 17 feet. The south vault will have a plan area of 1026 square feet and a maximum depth of 11 feet, with a maximum inside height of 7 feet.

Significant cuts and fills are likely for site grading and this work will be performed in conjunction with construction of retaining walls at various locations on the project site. As shown on the CORE Design drawings, in the northwestern part of the project, in the area of the access road and Lots 9/10, a retaining with a maximum height of about 9 feet is anticipated. This wall is expected to be primarily in a cut soil area. Along the south side of the access roadway, starting from about Lot 7 and extending to the east and south along the roadway edge, a wall with a maximum height of about 18 feet is planned. A retaining wall will also be constructed from the south edge of Lot 24 and extending to the south along the east edge of the access road, approximately to the northern detention vault. Both these walls will support structural fill soils. In the area of TP-7 and the looser silt soils, the new road grade will be in fills from 8 to 13 feet in depth.

The recommendations contained in the following sections of this report are based on our general understanding of design concepts provided by the project engineer. If changes are made, they should be reviewed by us and our recommendations modified, if necessary.

SCOPE OF WORK

We previously excavated eight test pits in the northern and eastern parts of the site, to determine the general soil conditions underlying the area of the property to be developed. In addition, two deep test borings were also drilled to depths of about 56 feet to evaluate the stability of the slopes on the site. The test pits were excavated on January 26, 2017 and were dug to a maximum depth of about 10.5 feet below the existing ground surface. The test borings were drilled on February 27, 2017. Most of the pits and the two borings encountered medium dense overburden sand to depths of upto five feet, followed by moderately dense glacially consolidated soils near the surface. Some deeper medium dense silty soils were noted in the lower project area in Boring B-2. Using the information obtained from the subsurface exploration, we developed geotechnical recommendations for project design and construction.

Specifically, this report addresses the following:

- Site Hazard Areas Evaluation
- Soil and groundwater conditions
- Slope Stability Study
- Geologic Hazards
- Site preparation and grading
- Foundation design
- Basement Walls
- Retaining Walls
- Detention Vault Walls and Foundations
- Slab-on-grade floors
- Infiltration Assessment
- Drainage
- Utilities
- Driveway pavements

It should be noted that the recommendations outlined in this report regarding drainage are associated with soil strength, design earth pressures, erosion and stability. Design and performance issues with respect to moisture and seepage as it relates to structure environment (i.e., humidity, mildew, and mold) are beyond the scope of our study. A building envelope specialist or contractor should be consulted to address these potential issues.

SITE CONDITIONS

Surface

The approximate site location is shown on the Vicinity Map, Figure 1. The general site topography and the currently proposed layout are shown on Figure 2, Exploration Location Plan. The 5.7 acre Emrick Property lies on sloping topography in the 7300 block of the Renton - Issaquah Road (State Highway 900) in Issaquah, Washington.

The irregular shaped rectangular site measures about 310 feet wide in a north-south direction and varies between 640 to 830 feet long east-west. An older residence has been demolished within the last few years and was located on a pad situated in the southeast corner of the property. Access to the elevated site is from an easement located on the adjacent property to the southeast. A long retaining wall along the west shoulder and sidewalk on the Renton-Issaquah Road right-of-way are located below the east site boundary.

The sloping site is located between the roadway below and the Talus Division 5C site above the upper western property line. A single- family residence lies northwest of the property, and a short plat containing several homes lies just north of the Emrick Property. The large parcel to the south is currently vacant.

The Emrick property is covered with forest, brush, and some grass areas. Several wetlands and a stream have been mapped on the site and are shown with their associated buffers on the site plan on Figure 2. The small stream flows from the northwest section of the site downslope in a southeasterly direction across the width of the property. A broad steep slope area is located on the upper west section of the property extending up to Division 5C of the Talus development. This division of Talus was developed with homes a number of years ago. Additional mapped steep slope areas exist along the eastern wall alignment above the highway, and wrapping around the northeast corner of the site. The adjacent short plat northeast of the site has mapped steep slope areas on it which extend over to the north site line in some areas.

The topography of the site is shown on the site plan on Figure 2. The topography of the property varies from gentle slopes up to steeper slopes, primarily on the western edge of the site.

The developable areas of the site have general topography in the 15 to 25 percent slope range. Site elevations vary from about Elevation 130 feet near the east line on top of the State Highway wall, up the west property line at about Elevation 310 feet.

Soil Exploration and Conditions

On January 26, 2017, we conducted part of our subsurface exploration by excavating eight test pits with a small trackhoe around the developable areas of the site. These pits were dug to depths of up to 10.5 feet below existing grades. The approximate test pit locations are shown on the Exploration Location Plan, Figure 2. The locations were approximately determined by pacing from known landmarks. An engineering geologist maintained a log of each test pit as it was excavated, classified the soil conditions encountered, and obtained representative soil samples. All soil samples were visually classified in accordance with the Unified Soil Classification System.

To provide supplemental information of some of the deeper soils conditions, we drilled two test borings to depths of up to 56 feet. The borings were drilled by Holocene Drilling using a small tracked drill rig on February 27, 2017. The approximate boring locations are also shown on the Exploration Location Plan, Figure 2. The drill locations were approximately determined by pacing from known landmarks. An engineering geologist maintained a log of each boring as it was drilled. During drilling, soil samples were obtained in general accordance with ASTM Test Designation D-1586.

Using this procedure, a two-inch (outside diameter) split barrel sampler is driven into the ground 18 inches using a 140-pound hammer free falling a height of 30 inches. The number of blows required to drive the sampler 12 inches after an initial 6-inch set is referred to as the Standard Penetration Resistance value or "N" value. This is an index related to the consistency of cohesive soils and relative density of cohesionless materials. "N" values obtained for each sampling interval are recorded at the appropriate depths on the boring logs attached to this report.

Representative soil samples obtained from the test pits and the borings were placed in sealed plastic bags and jars and returned to a laboratory for further examination. Several moisture content determinations were made on samples and the results are listed on the appropriate test pit and boring logs. The test pit and the boring logs are presented with this report as Figures 3 through 12.

The near surface soils encountered in the test pits and the borings are very similar in most areas of the site. Below several feet of weathered silty sand and sandy silt residual soil, we encountered moderately dense to dense glacially consolidated till – like soils near the ridge top areas of the planned home locations. All the test pits and borings other than TP-7, encountered these relatively dense and consolidated silty sands and sandy silts with pebbles in the upper fifteen feet or so. The till-like denser soils in Test Pits 1,2 ,3 ,4 5, 6, and 8 were found at depths of 2 to 5 feet in depth. Some of the dense till-like soil was mapped as Silty Till as was found in the pit near the planned north detention vault at TP-8.

Test Pit TP-7 is located near one of the interior wetlands and near the proposed new access street. This test pit encountered more than 10 feet of stiff to very stiff, non-stratified clayey Silt. We anticipate that this moderately plastic soil is an ice contact pond deposit which is discussed below.

No significant old fills were found in the test pits other than around the demolished house pad at TP-8. We anticipate that some minor fills exist around this house site.

The borings encountered a variety of deeper soils under the near surface relatively dense till-like consolidated soils. The upper boring (B-1) which is closer to the steep western hillside encountered denser soils for the full depth of the boring. Dense to very dense interbedded silty sands, pebbly sandy silts, and clean gravelly sand layers were found at this location. These soils were all glacially consolidated under some valley glacier loading. The lowermost unit consisting of clayey sandy silt with pebbles is also densely consolidated.

The lower boring (B-2) encountered about 10 to 15 feet of dense consolidated till-like soils consisting of pebbly sandy silt and silty sand. These soils graded into a massive unit of possibly ice contact silt or silty ablation till which was not densely consolidated by glacial action. This clayey sandy silt layer extended to the full depth of the boring. This soil layer was not bedded or jointed and was massive and undisturbed. Below 45 feet, it became somewhat denser. This layer of more clayey sandy silt appears similar to the soil unit found below a depth of 45 feet in the upper boring. However, the soil in the upper boring was consolidated by glacial action to a very dense state.

The glacial till-like soils were probably deposited by the last ice age glacier passing over the area 12,000 to 15,000 years ago. The till is composed of a mix of silt-sand-gravel and densely consolidated by glacial action. The site area is on the west wall of the Tibbets Creek valley south of Issaquah. In this area of Issaquah, we believe that the till was "plastered" on the side-hill over the lower sections of the slope below the Talus development. The till probably overlies dense outwash soils as found in Boring B-1 interbedded by other consolidated ice-contact soils. Less consolidation of the soils was observed in the plastered till -like soils on the lower section of the site above the roadway at Boring B-2. This may be due to the glacial ice not being in place for the same length of time for consolidation as was noted further up the hill at B-1.

During the end of the last Ice Age, about 10,000 to 11,000 years ago, the stagnant remains of a valley glacier existed in the Tibbets and Issaquah Creek valleys. The glacier had deposited till-like soils along the sides of some of the valleys, as well as Ice Contact drift on the sidewalls of the canyons.

The ice contact drift consists of pond deposits of clay and silt, outwash sands and gravels from streams, and some interbeds of consolidated till. As the ice stagnated and melted, streams and ponds existed along the ice margins against the hillsides. Deposits of glacial drift then slumped down or were deposited on slopes as glacial margin deposits as the ice melted away from the valleys. Ice Contact drift has been mapped up and down these side hill slopes in the Tibbets and Issaquah Creek valleys.

The test pit and boring logs presented on Figures 3 through 12 present more detailed descriptions of the subsurface conditions encountered in the exploration.

The *US Soil Conservation Service Map for King County, Washington* shows the soils in the vicinity of the site are mapped as Alderwood glacial till soils in the lower two-thirds of the site. The higher western slope areas into the Talus development are mapped as Ragnar/Indianola glacial outwash soils. The more accurate *Geologic Map of the Issaquah Quadrangle, 7.5 Minute Map (By Booth and Minard, 1992)* indicates that most of the site is underlain by the Alderwood Glacial Till soils plastered on the lower slope areas of the valley.

Surrounding sections of Talus are mapped as being underlain by Ice Contact drift areas on the higher slopes around the site. This map also indicates that some higher areas into the Talus development are underlain by Tertiary age sandstone bedrock. A geotechnical report prepared by Golder Inc. for Talus Division 5 directly above the Emrick site's steep western slopes indicates that the slope is underlain at shallow depths by this same sandstone bedrock.

Groundwater

No significant groundwater was encountered in the test pits during excavation. The deeper borings indicate that groundwater seepage exists at depths of 14 to 18 feet beneath the surface. The nearby creeks and wetlands probably allow water to seep into the deeper subsurface layers. Some of the deeper outwash layers noted in Boring B-1 probably also transmit some groundwater laterally. These layers of outwash with seepage were not found in the lower Boring B-2. Seasonal fluctuations of groundwater levels will occur at this site. The looser near surface soils overlying the deeper till-like materials and the silt will tend to perch groundwater seepage during the wet winter months over the low permeability glacial till-like and silt soils. The perched groundwater gradient will be controlled by the subsurface topography of the top of the till and silt layers and the ground topography. In general, this perched winter seepage will flow downslope to the east and the southeast, following the topography. Some of this subsurface groundwater will be controlled by the western stream and wetland areas above the developable areas on the site. We anticipate that no significant groundwater seepage will occur during the dry summer months. Minor fluctuations in the seepage water levels at the site would also be expected following periods of heavy precipitation.

GEOLOGIC HAZARDS AND CRITICAL AREAS

Coal Mine Hazards

There are no known coal mine hazards on the site or in the vicinity.

Seismic Hazards

Based on our site-specific findings, the area of the site should not be classified as a Seismic Hazard Area. Based on the soil conditions encountered and the local geology, per Chapter 16 of the 2012 International Building Code (IBC), Site Class "D" should be used in structural design.

The following parameters should be used in structural design, as needed:

Seismic Design Parameters (IBC 2012)

Spectral Response acceleration (Short Period), Ss	**1.329**
Spectral Response acceleration (1 - Second Period), S1	**0.502**
Site Coefficient, Fa	**1.00**
Site Coefficient, Fv	**1.500**
Five percent damped .2 second period, Sds	**0.886**
Five percent damped 1.0 second, Sd1	**0.502**

Liquefaction is a phenomenon where there is a reduction or complete loss of soil strength due to an increase in water pressure induced by vibrations. Liquefaction mainly affects geologically recent deposits of fine-grained sands that are below the groundwater table. Soils of this nature derive their strength from inter-granular friction. The generated water pressure or pore pressure essentially separates the soil grains and eliminates this inter-granular friction; thus, eliminating the soil's strength.

Due to the dense glacial till-like soils under the site and the high fines content of the lower soils, it is our opinion that there is no significant risk for liquefaction to occur at this site, below the residences, during an earthquake.

Erosion Hazards

The site near-surface soils on the part of the site to be developed are mostly all residual glacial sediments consisting of weathered till, silty sand and some bedrock/till soils at depth. The developable sections of the plat lie on gentle to moderate sloping grades. We anticipate that the steeper areas on the property will not be disturbed. The till-like soils have a low to moderate erosion hazard potential based on the existing shallow slope gradients but are typically developed upon for similar projects in the Puget Sound basin.

Best Management Practices (BMPs) should be used during construction to mitigate the potential erosion hazard. If the erosion control measures are properly implemented and maintained, it is our opinion that the planned residential development will not adversely impact the erosion potential for the site or adjacent properties.

As a minimum, we recommend implementing erosion and sediment control BMPs prior to, during, and immediately following clearing and grading activities at the site. Application of BMPs should conform to the standards and specifications presented in the King County Storm Water manual as well as City of Issaquah requirements.

Steep Slope/Landslide Hazards

Generally, the lot areas slope gently down to the east and the southeast over gradients of 15 to 25 percent slopes.

The development plan indicates that a 15 foot wide buffer will be used in most areas between the back east and northeast lot lines and the top of the steeper 40 percent slopes shown on the development plan. These slopes are located behind Lots 20 through 24 on the development. Based on the dense consolidated soil types, lack of groundwater or springs, it is our opinion that there is no significant Landslide Hazard risk on the subject property based on the presence of dense glacial till – like soils at shallow depth.

This risk from this potential hazard is further mitigated by the setback of construction slope disturbance and the siting of new homes behind the recommended slope setback/buffer.

Slope Setbacks

For Lots 19 to 24, the house locations are setback more than 25 feet from the top of the 40 percent slopes on the Emrick property, in accordance with our recommendations. On Lot 24, the minimum setback appears to be on the order of about 20 feet. On this lot, the foundations may be deepened to provide a 25-foot setback. On all lots, the setback distance maybe modified slightly if the foundations on the slope side are deepened. We will be pleased to assist you in determining appropriate modifications and depths once the final building layouts have been prepared.

Regardless of building setbacks, structures such as decks and/or patios may be constructed in the setback zone. These should be setback at least 10 feet from the top of the slope and should be structurally separated from the residences.

SLOPE STABILITY ANALYSES

We examined the potential for various possible modes of failure.

We conducted analyses for two cross-sections. The first was Section A-A' which extends downhill from near Lot 24 down to the level of the Renton Issaquah Highway. The second was Section B-B' which extends up from behind Lot 7 to Talus Division 5 above. The soil profile and the soil parameters used in our analyses were based on the results of our subsurface exploration and those presented in a report for Talus 5 prepared by Golder Associates and dated June 5, 2000. In general, both sets of exploration show three to six feet of overburden soils consisting of medium dense to dense silty sands.

Below, dense glacial till or till-like soils and/or bedrock are present. Groundwater was not observed in most of the explorations, however, the graphic results of the analyses show the level used in the analysis. The profiles and the soil parameters used are shown on Figures 13 and 14.

We performed our stability analyses using the computer program WINSTABL which was developed by the University of Wisconsin.

We analyzed the slope stability for both static and pseudostatic seismic conditions. Based on published data from the USGS, the peak ground acceleration for this area is 0.33g. In conducting the stability analysis for seismic conditions, we used a co-efficient of 0.17g (half the peak acceleration) in conformance with current geotechnical practice.

The results of the slope stability analyses are presented on figures in Appendix A, with each figure showing the 10 most critical failure surfaces for static and seismic conditions and the safety factors for each condition. As shown on the slope stability worksheets (Appendix A), the most critical failure surfaces have a moderate depth and extend virtually the full height of the slope.

The results of the analyses are summarized below:

SUMMARY OF SLOPE STABILITY ANALYSES

Section Analyzed	Lowest Safety Factor(static)	Lowest Safety Factor (Seismic)
Section A-A′	3.84	2.38
Section B-B′	1.94	1.38

Current practice requires a minimum safety factor of 1.5 for static conditions and a minimum safety factor of 1.1 for seismic conditions. Based on these criteria, the safety factors are adequate from the standpoint of overall slope stability. However, these numbers should not be taken as absolute values. They are generally representative of the slope stability conditions at the sections analyzed and indicate overall adequate stability.

DISCUSSION AND RECOMMENDATIONS

General

Based on our study, the site is suitable for the proposed development. The planned residences can be supported on conventional spread footings bearing on competent native soils below the topsoil and any localized old fill.

If required, spread footings can also be supported on structural fill or clean rock placed and compacted above the competent native soils. Garage and daylight basement floor slabs and pavements can be similarly supported.

Due to the sloping nature of the site, some of the new homes will probably have daylight basements which will need to be designed as retaining walls. Considering the likelihood of a perched groundwater developing in the winter months, the walls must be provided with adequate subsurface drainage.

Retaining walls upto 18 feet high will also be requited to support the access roadway and the slope north and northwest of the wetland. These walls will be in cut and fill soils and must be appropriately designed and constructed. Details are presented later in this report.

All fills must be properly compacted. Materials obtained from onsite excavations are likely to have a high percentage of fines and likely may not be usable for structural fill during wet weather.

The following sections provide detailed recommendations regarding the above issues and other geotechnical design considerations. These recommendations should be incorporated into the final design drawings and construction specifications.

Site Preparation and Grading

To prepare the site for construction, all vegetation, organic surface soils, and other deleterious materials should be stripped from below the access driveways and house building areas.

Soils containing organic material will not be suitable for use as structural fill, but may be used in non-structural areas or for landscaping purposes.

Some of the shallow on-site soils generally appear suitable for use as structural fill in the drier summer months only. However, the silty sands, weathered tills and the site silts may be difficult to use as fill or backfill during some periods of the year. The ability to use these upper silty sands from site excavations as structural fill will depend on their moisture content and the prevailing weather conditions at the time of construction.

If grading activities must take place during wet weather or in the wet season, the owner should be prepared to use wet weather structural fill as needed. For this purpose, we recommend using a granular soil which meets the following grading requirements:

Maximum Aggregate Size	3 inches
Minimum Retained on the No. 4 Sieve	25 percent

Maximum Passing the No. 200 Sieve	5 percent*

*Based on the 3/4 inch fraction

Prior to use, the geotechnical engineer should examine and test all on-site or imported materials proposed for use as structural fill. Alternatively, railroad ballast or clean small quarry spalls may be used over wet subgrades or in any old utility area over-excavations as structural fill material.

Structural fill should be placed in uniform loose layers not exceeding 12 inches and then compacted to a minimum of 95 percent of the soil's maximum dry density as determined by ASTM Test Designation D-698 (Standard Proctor). The moisture content of the soil at the time of compaction should be within two percent of its optimum, as determined by this ASTM standard. In non-structural areas or for backfill in utility trenches below a depth of four feet, the degree of compaction could be reduced to 90 percent.

Excavations and Slopes

All excavations at the site associated with confined spaces, such as utility trenches, must be completed in accordance with local, state, or federal requirements. Based on current Occupational Safety and Health Administration (OSHA) regulations, the on-site soils would be generally classified as Group B soils.

Temporary excavations should be made no steeper than 1:1 (H:V). Permanent cut or fill slopes should not be steeper than 2:1. All permanent slopes should be planted with suitable species of vegetation to minimize the potential for erosion.

Caving of trench sidewalls in the upper loose soils may occur in some trench areas where temporary excavations are deeper than about four feet. Below a depth of about three to five feet, the partially to fully cemented soil layers of till-like materials and hard silt may exist in most areas. Temporary excavations cut in this material can be made at a slope of 0.75:1 (H:V). The contractor should use appropriate safety precautions and trench boxes, as needed.

While the test pits generally did not encounter any groundwater at the time of excavation, we expect that some groundwater seepage in excavations extending below the perched winter seepage level may occur based on the depth of the planned utilities.

Dewatering should be anticipated if excavations will extend below the perched seepage level in the wet months of the year. The amount of water flow into excavations will be relatively low. Conventional sump pumping procedures, along with a system of collection trenches, if necessary, should be capable of maintaining a relatively dry excavation for construction purposes.

The above information is provided solely for the benefit of the owner and other design consultants, and should not be construed to imply that ABPB Consulting assumes any responsibility for job site safety. It is understood that job site safety is the sole responsibility of the project contractor.

Foundations

The new house foundations may be supported on conventional spread footings bearing on competent native soils, or on structural fills placed above competent native soils.

Foundation subgrades should be prepared as recommended in the Site Preparation and Grading section. Perimeter foundations should extend at least 1.5 feet below final exterior grades. Foundations on the downhill sides of the houses on or near slopes should be at a minimum depth of 24 inches below the lowest adjacent finish grade. Interior foundations can be constructed at any convenient depth. All footing excavations should be thoroughly re-compacted with hand equipment following excavation disturbance.

House footings should have a minimum width of 15 inches or be in accordance with the IBC standards. We recommend designing foundations for a net allowable bearing capacity of 2,500 pounds per square foot (psf). Foundations for the anticipated deeper vaults can utilize a higher bearing capacity of 5,000 pounds per square foot

For short-term loads, such as wind and seismic, a 1/3 increase in these allowable capacities can be used. For the anticipated loads and bearing stresses, estimated total settlements should be 1/2 to 3/4 inch, of which 1/4 to 1/2 inch would be differential.

For designing foundations to resist lateral loads, a friction coefficient of 0.4 can be used. Passive earth pressures acting on the sides of the footings and buried portions of the foundation stem walls can also be considered. We recommend calculating this lateral resistance using an equivalent fluid weight of 350 pounds per cubic foot (pcf).

We recommend not including the upper 12 inches of soil in this computation because they can be affected by weather or disturbed by future grading activity. This value assumes the foundations will be constructed neat against competent soil or backfilled with structural fill as described in the Site Preparation and Grading section. The values recommended include a safety factor of 1.5.

Basement and Vault Retaining Walls

It is anticipated that some retaining walls will be needed for daylight basement homes, as well as the retaining walls for the detention vaults. The magnitude of earth pressures developing on any proposed retaining walls will depend on the quality of the wall backfill. We recommend placing and compacting wall backfill as structural fill.

Below improved areas such as pavements or floor slabs, the backfill should be compacted to a minimum of 95 percent of its maximum dry unit weight, as determined by ASTM Test Designation D-698 (Standard Proctor). In unimproved areas, the relative compaction can be reduced to 90 percent. To prevent hydrostatic pressure development, wall drainage must also be installed.

With wall backfill placed and compacted as recommended and drainage properly installed, we recommend designing unrestrained walls for an active earth pressure equivalent to a fluid weighing 35 pcf. For restrained walls, an additional uniform lateral pressure of 100 psf should be added. These values assume a horizontal backfill condition and that no other surcharge loading, such as traffic or sloping embankments, will act on the wall. If such conditions will exist, then the imposed loading must be included in the wall design. For seismic loading conditions, a uniform pressure of 7H psf should be added, where H is the wall height.

Friction at the base of foundations and passive earth pressure will provide resistance to these lateral loads. Values for these parameters are provided in the Foundations section.

For detention vault wall foundations, an allowable bearing pressure of 5000 pounds per square foot may be used.

Access Roadway Retaining Walls

Walls upto 18 feet high which will retain fills may be required for supporting the access roadway and the slope to the northwest, north, and to the east of the wetland.

The slope adjacent to Lot 10 and extending down along the access roadway to Lot 9 will be in a cut area and have a maximum height of about 9 feet. It is expected that the upper five feet of soils excavated in this area will be medium dense overburden silty sands, then underlain by weathered to unweathered glacial till or till-like materials. In areas where the height of the wall is less than 5 feet high, a rockery constructed in accordance with ARC standards may be used. Where wall heights are more than 5 feet, the wall may be constructed as a structural concrete wall, an MSE (mechanically stabilized earth) or a reinforced rockery. Parameters for design of these walls are presented below.

Along the access roadway around the wetland margin, the walls will be supporting structural fill soils. We recommend that these walls be constructed as MSE walls such as reinforced earth, keystone or other similar walls. Design parameters are given below:

Angle of internal friction of Soil = 34 degrees

Soil Cohesion = 200 psf

Unit weight of soil = 120 pcf.

Regardless of the type of wall used, it is imperative that it be provided with adequate subsurface drainage to prevent the development of hydrostatic pressures.

It has been our experience that MSE walls be built under a design-build contract for maximum efficiency. We will be pleased to provide any additional information needed to facilitate this process.

Slab-on-grade Floors

We anticipate that some slabs-on-grade will be provided within the house garage areas and or in the lower level of any daylight basement homes. .Slab-on-grade floors may be supported on the subgrade prepared as recommended in the Site Preparation and Grading section.

If moisture intrusion is a concern, the slabs should be provided with a four-inch thick capillary break layer of clean, free-draining sand or gravel that has less than three percent fines passing the No. 200 sieve. This material will reduce the potential for upward capillary movement of water through the underlying soil and subsequent dampening of the garage floor slab. Where moisture by vapor transmission is undesirable, a durable plastic membrane should be placed below the slab above the capillary break.

This membrane is commonly covered with one to two inches of clean, moist sand to protect damage during construction and to aid in curing of the concrete. Other methods are available for preventing or reducing water vapor transmission through the slab. We recommend consulting with a building envelope specialist for additional assistance regarding this issue.

Drainage

Storm Water Infiltration Potential

The soils observed in the test pits are dense glacial till-like or Silt soils beneath depths of two to three feet. These soils have a significant percentage of "fines" and have a low permeability. These soils will not allow any significant infiltration into the ground surface due to the finer grained matrix of soil and the likely presence of an elevated winter water table. These soils can be classified as a Silt Loam type for infiltration design. The anticipated design infiltration rate for these soils is less than 1 inch per hour.

Surface

Final exterior grades should promote free and positive drainage away from the house areas at all times. Water must not be allowed to pond or collect adjacent foundations or within the immediate building areas or be allowed to flow over the top of any of the site slope areas.

We recommend providing a gradient of at least three percent for a minimum distance of five feet from the building perimeters, except in paved locations. In paved locations, a minimum gradient of one percent should be provided, unless provisions are included for collection and disposal of surface water adjacent the structures.

Subsurface

We recommend installing continuous drains along the outside lower edge of the perimeter house foundations and foundation basement walls. The foundation drains and roof downspouts should be tight-lined separately to approved discharge facilities. Subsurface drains must be laid with a gradient sufficient to promote positive flow to a controlled point of approved discharge. Lower level drainage should be installed as noted above at the level of the lowest wall footings.

All drains should be provided with cleanouts at easily accessible locations. These cleanouts should be serviced regularly.

Utilities

Utility pipes should be bedded and backfilled in accordance with American Public Works Association (APWA) and City of Issaquah specifications. As a minimum, trench backfill should be placed and compacted as structural fill as described in the Site Preparation and Grading section.

We suggest that utility lines not be placed over 40 percent sloping areas without special construction procedures being utilized. Backfill should be thoroughly compacted in all areas. If the soils excavated on-site are free of excessive deleterious material or debris and are not excessively moist, they can be suitable for use as backfill material during the drier months. If construction takes place during winter or spring, it may be necessary to import structural fill for backfilling purposes.

If proposed elevations of buried utilities will extend beneath the perched water table, dewatering may be necessary and excavations may need to be provided with temporary shoring support.

Access Driveways and Pavements

Driveway pavements and the planned road for the plat should be constructed on subgrades prepared as described in the Site Preparation and Grading section. However, regardless of the relative compaction achieved, the subgrade must be in a firm and relatively unyielding condition prior to paving. The subgrade should be proof-rolled with heavy construction equipment to verify this condition.

The appropriate pavement section depends upon the supporting capability of the subgrade soils and the traffic conditions to which it will be subjected. We expect that traffic will mainly consist of light passenger vehicles, construction concrete trucks and commercial vehicles in the form of trash removal vehicles.

Based on the above information, with a stable subgrade prepared as recommended, we recommend the following pavement sections for light automobile traffic:

- Two inches of asphalt concrete (AC) over six inches of crushed rock base (CRB)
- Two inches of AC over four inches of asphalt treated base (ATB)

The paving materials used should conform to the Washington State Department of Transportation (WSDOT) specifications for Class B asphalt concrete, ATB, and CRB.

Long-term pavement performance will depend on surface drainage. A poorly-drained pavement section will be subject to premature failure as a result of surface water infiltrating into the subgrade soils and reducing their supporting capability. To improve pavement performance, surface drainage gradients of no less than two percent are recommended. Also, some longitudinal and transverse cracking of the pavement surface should be expected over time. Regular maintenance should be planned to seal cracks when they occur.

The following figures are included and complete this report:

Figure 1	Vicinity Map
Figure 2	Exploration Location Plan
Figures 3 through 10	Test Pit Logs
Figures 11 and 12	Boring Logs
Figures 12 and 13	Sections A-A′ and B-B′
Appendix A	Slope Stability Analysis

We prepared this report in accordance with generally accepted geotechnical engineering practices. No other warranty, expressed or implied, is made. This report is the property of ABPB Consulting and is intended for specific application to the Emrick Property residential project in Issaquah, Washington. This report is for the exclusive use of Burnstead Construction and their authorized representatives.

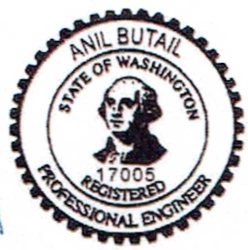
We appreciate the opportunity to be of service during this phase of project development and design and look forward to working with you during the final design and construction phases.

We trust the information presented in this report is sufficient for your current needs. If you have any questions or need additional information, please call.

Sincerely yours,

ABPB CONSULTING

Paul K. Bonifaci
Project Engineering Geologist

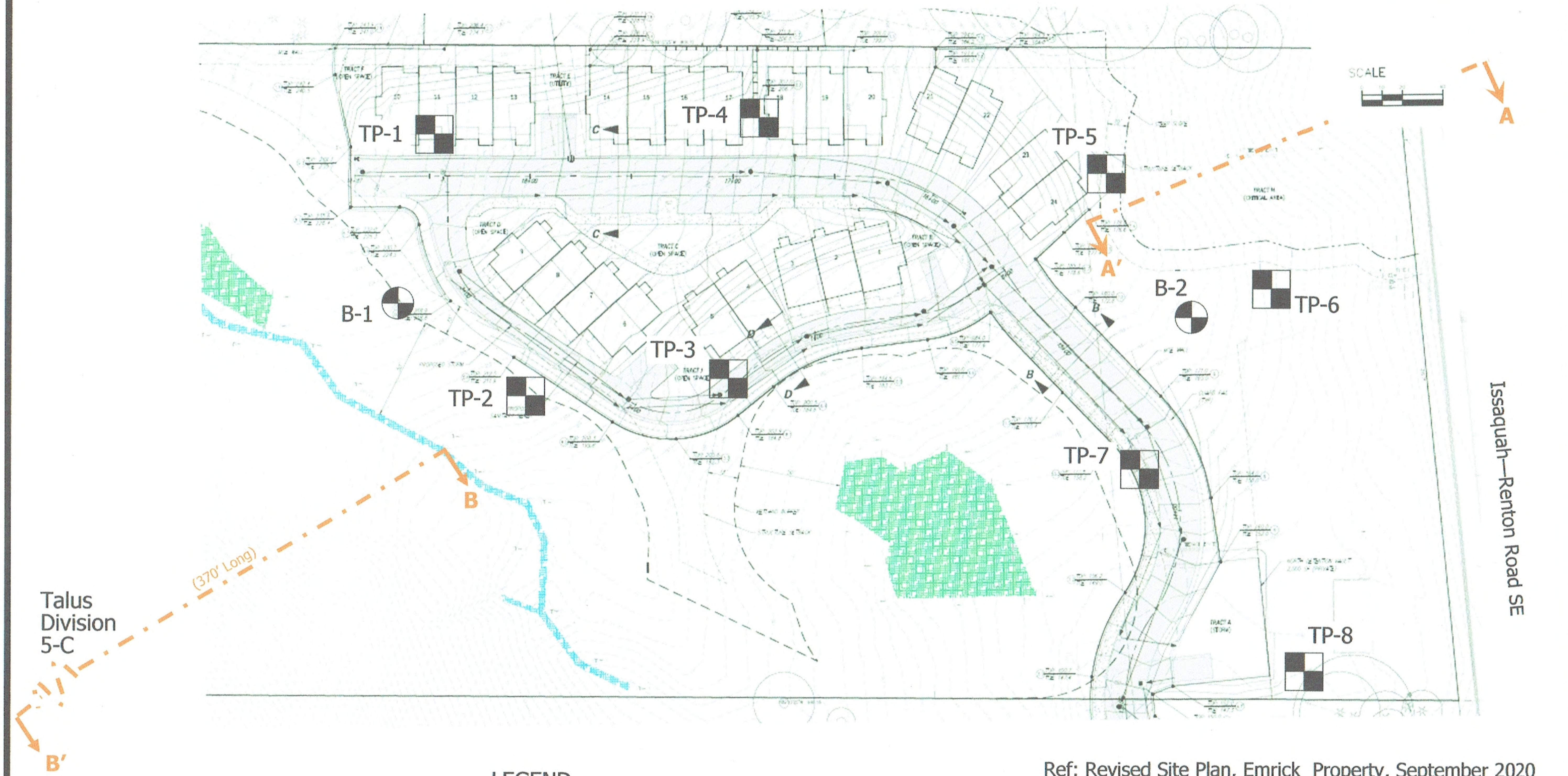


Anil Butail, P.E.
Principal Engineer

NTS

Ref: Windows Air Photos

ABPB Consulting Geotechnical Consultants Kirkland, Wash.	Vicinity Map Emrick Property Issaquah, Washington		
	Proj. No. 1455	Date : 2-17	Figure 1



LEGEND

TP-1 Approx. Test Pit Location and Number

B-1 Approx. Boring Location and Number

Approx. Location of Cross Sections

Ref: Revised Site Plan, Emrick Property, September 2020

ABPB Consulting
Geotechnical Consultants
Kirkland, Wash.

Site Exploration Plan
Emrick Property
Issaquah, Washington

Proj. No. 1455	Date : Sept. 2020	Figure 2

Project :	Emrick Property			Test Pit TP - 1
Project No.	1455	Date :	1-26-17	
Client :	Burnstead	Elevation	230 feet	
Location:	Issaquah	Logged By:	PKB	

SUBSURFACE PROFILE			SAMPLE			Field Strength Tests	Laboratory Results
Depth (ft)	Soil Lithology	Soil Description	Water Level	Sample	USCS		Moisture Content
0 to -5		Silty Sand: (13 inches Sod/Fill) Red tan to Tan, silty Sand with gravel, silt and roots, medium dense, moist			SM		
-5 to -9.5		Silty Sand: Mottled tan grading to gray tan, pebbly silty SAND to sandy SILT, dense with depth, moist (Weathered to unweathered Glacial Till) Test Pit terminated at 9.5 feet. No groundwater encountered.		(sample at -6 to -7)	SM		23.0%

Project :	**Emrick Property**		
Project No.	1455	**Date :**	1-26-17
Client :	Burnstead	**Elevation**	206 feet
Location:	Issaquah	**Logged By:**	PKB

Test Pit TP - 2

Depth (ft)	Soil Lithology	Soil Description	Water Level	Sample	USCS	Field Strength Tests	Moisture Content
0 to -4		Silty Sand: (13 inches Sod/Fill) Red tan to Tan, silty Sand with gravel and some cobbles, some roots, medium dense, moist			SM		
-4 to -6		Silty Sand: Mottled tan grading to gray tan, pebbly silty SAND to sandy SILT, medium dense to dense, moist (Weathered Glacial Till)			SM		
-6 to -9.7		Silty Sand: Tan grey, silty SAND to sandy SILT, with pebbles, dense grading very dense, moist (Glacial Till) Test Pit terminated at 9.7 feet No groundwater seepage encountered			SM		23.4%

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Project :	Emrick Property			Test Pit TP - 3
Project No.	1455	Date :	1-26-17	
Client :	Burnstead	Elevation	190 feet	
Location:	Issaquah	Logged By:	PKB	

SUBSURFACE PROFILE			SAMPLE			Field Strength Tests	Laboratory Results
Depth (ft)	Soil Lithology	Soil Description	Water Level	Sample	USCS		Moisture Content
0 to -4.5		Silty Sand: (9 inches Sod/Fill) Red tan to Tan, silty Sand with gravel and some cobbles, some sandy Silt and roots, medium dense, moist			SM		
-4.5 to -6.5		Sandy Silt: Mottled tan grading to gray tan, pebbly sandy SILT, medium dense to dense, non-plastic, moist (Weathered Glacial Till)		Sample (-5 to -6)	ML		15.4%
-6.5 to -9.1		Sandy Silt: Tan grey, sandy SILT, with pebbles, dense grading very dense, non-plastic, moist (Glacial Till) Test Pit terminated at 9.1feet No groundwater seepage encountered		Sample (-7 to -8)	ML		20.7%

Project :	**Emrick Property**			Test Pit TP - 4
Project No.	1455	**Date :**	1-26-17	
Client :	Burnstead	**Elevation**	206 feet	
Location:	Issaquah	**Logged By:**	PKB	

SUBSURFACE PROFILE			SAMPLE			Field Strength Tests	**Laboratory Results**
Depth (ft)	Soil Lithology	Soil Description	Water Level	Sample	USCS		Moisture Content
0 to -2		Silty Sand: (9 inches Sod/Fill) Red tan to Tan, silty Sand with gravel, silt and some roots, medium dense, moist			SM		
-2 to -6		Silty Sand: Mottled tan grading to gray tan, pebbly silty SAND to sandy SILT, medium dense grading to dense with depth, moist (Weathered Glacial Till)			SM		
-6 to -9		Silty Sand: Tan grey, pebbly silty SAND with some sandy Silt, dense to very dense, moist (Glacial Till) Test Pit terminated at 9 feet No groundwater encountered		(sample -6 to -7)	SM		10.2%

Project :	Emrick Property			Test Pit TP - 5
Project No.	1455	Date :	1-26-17	
Client :	Burnstead	Elevation	172 feet	
Location:	Issaquah	Logged By:	PKB	

SUBSURFACE PROFILE			SAMPLE			Field Strength Tests	Laboratory Results
Depth (ft)	Soil Lithology	Soil Description	Water Level	Sample	USCS		Moisture Content
0 to -4		Silty Sand: (14inches Sod/Fill) Red tan to Tan, silty Sand with gravel, some cobbles, silt and some roots, medium dense, moist			SM		
-4 to -8.5		Silty Sand: Mottled tan grading to gray tan, pebbly silty SAND with some slightly silty gravelly Sand, dense grading to very dense with depth, moist to damp (Glacial Till) Test Pit terminated at 8.5 feet No groundwater encountered			SM		

Depth scale: 0, -1, -2, -3, -4, -5, -6, -7, -8

Project :	**Emrick Property**		
Project No.	1455	**Date :**	1-26-17
Client :	Burnstead	**Elevation**	152 feet
Location:	Issaquah	**Logged By:**	PKB

Test Pit TP - 6

SUBSURFACE PROFILE			SAMPLE				**Laboratory Results**
Depth (ft)	Soil Lithology	Soil Description	Water Level	Sample	USCS	Field Strength Tests	Moisture Content
0		Silty Sand: (9 inches Sod/Fill) Red tan to Tan, silty Sand with gravel, and some roots, medium dense, moist					
-1					SM		
-2		Sandy Silt: Mottled tan grey grading to gray , pebbly sandy SILT, some gravel silty Sand, non-plastic, dense and hard, moist (Silt Glacial Till)					
-3							
-4					SM		21.4%
-5		Test Pit terminated at 8.0 feet No groundwater encountered			ML		
-6							
-7							
-8							

Project :	Emrick Property		
Project No.	1455	**Date :**	1-26-17
Client :	Burnstead	**Elevation**	154 feet
Location:	Issaquah	**Logged By:**	PKB

Test Pit TP - 7

SUBSURFACE PROFILE			SAMPLE			Field Strength Tests	**Laboratory Results**
Depth (ft)	Soil Lithology	Soil Description	Water Level	Sample	USCS		Moisture Content
0 to -2		Silty Sand: (9 inches Sod/Fill) Red tan to Tan, silty Sand with some gravel and sandy Silt, and some roots, medium dense, moist			SM		
-2 to -9		Sandy Silt: Mottled orange and tan, sandy clayey SILT, low to medium plasticity, stiff, moist to damp					
-5				Sample	ML		32.5%
-6		Test Pit terminated at 9.0 feet Light seepage at 2 and 5 feet					
-8							39.1%

Project :	**Emrick Property**		
Project No. 1455		**Date :**	1-26-17
Client :	Burnstead	**Elevation**	146 feet
Location:	Issaquah	**Logged By:**	PKB

Test Pit TP - 8

SUBSURFACE PROFILE			SAMPLE			Field Strength Tests	Laboratory Results
Depth (ft)	Soil Lithology	Soil Description	Water Level	Sample	USCS		Moisture Content
0 to -2.5		Fill: (2 inches Sod/Fill) Red tan to Tan, silty Sand with gravel, loose, moist (Old house backfill)			SM		
-2.5 to -5		sillty Sand: Red tan, silty Sand with some roots and gravels, loose to medium dense, moist			SM		
-5 to -10.5		Silty Sand: Tan to tan grey, layers of pebbly silty Sand, and non-plastic sandy Silt, hard to dense, moist (Till-like)			SM ML		
-8 to -9		Test Pit terminated at 10.5 feet No groundwater encountered		Sample			13.2%

Depth scale: 0, -1, -2, -3, -4, -5, -6, -7, -8, -9, -10

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Project :	Emrick Property		
Project No.	1455	**Date :**	2-27-17
Client :	Burnstead	**Elevation**	224 feet
Location:	Upper Plat Area	**Logged By:**	Paul Bonifaci

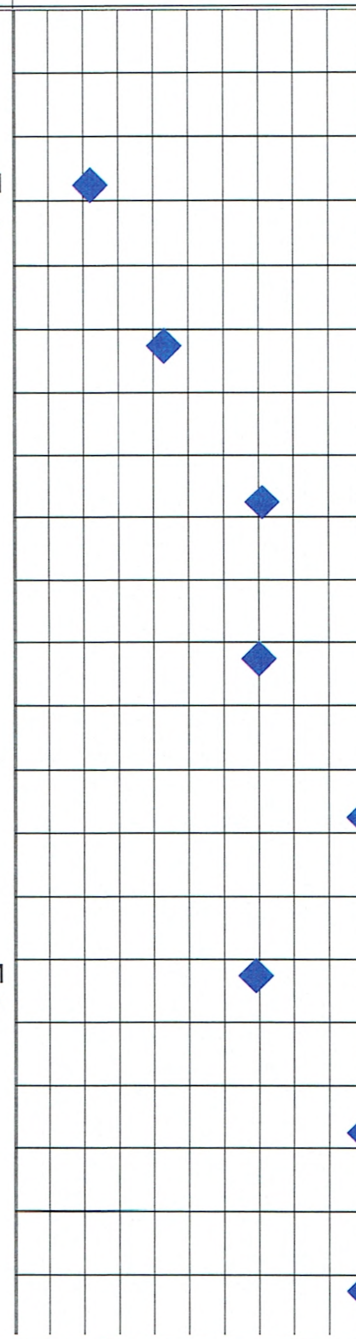
Boring No. B-1

SUBSURFACE PROFILE: Depth (ft)	Soil Description	USCS
0 to -28	Silty Sand: Tan to grey, partially to cemented layers of gravelly silty Sand (till-like) to pebbley sandy Silt, numerous layers of damp clean gravelly Sand, moist grading wet, dense grading to very dense, non-bedded and non-jointed	SM ML
-28 to -42	Sand: Grey tan layers of mostly slightly silty to clean gravelly Sand, damp to wet, very dense with occasional pebbly sandy Silt (till-like), damp to moist, very dense (Outwash)	SM SP

Water Level: ▼ at approx. -18 ft

△ Standard Penetration Resistance (SPT)

Blows/Foot: 10, 50, 100



Laboratory Results

Moisture Content

14.4%

11.1%

9.4%

17.2%

11.3%

11.8%

13.0%

10.4%

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Project :	**Emrick Property**		
Project No.	1455	**Date :**	2-27-17
Client :	Burnstead	**Elevation**	224 feet
Location:	Upper Plat Area	**Logged By:**	Paul Bonifaci

Boring No. B-1

SUBSURFACE PROFILE			SAMPLE			△ Standard Penetration Resistance (SPT) Blows/Foot 10 50 100	**Laboratory Results** Moisture Content
Depth (ft)	Soil Lithology	Soil Description	Water Level	Sample	USCS		
-42 to -50		Sandy Silt: Blue grey, clayey silty Sand to clayey sandy SILT, very dense to hard, moist, low plasticity, Groundwater below 18 feet. Boring terminated at 50.5 feet			ML		15.1%

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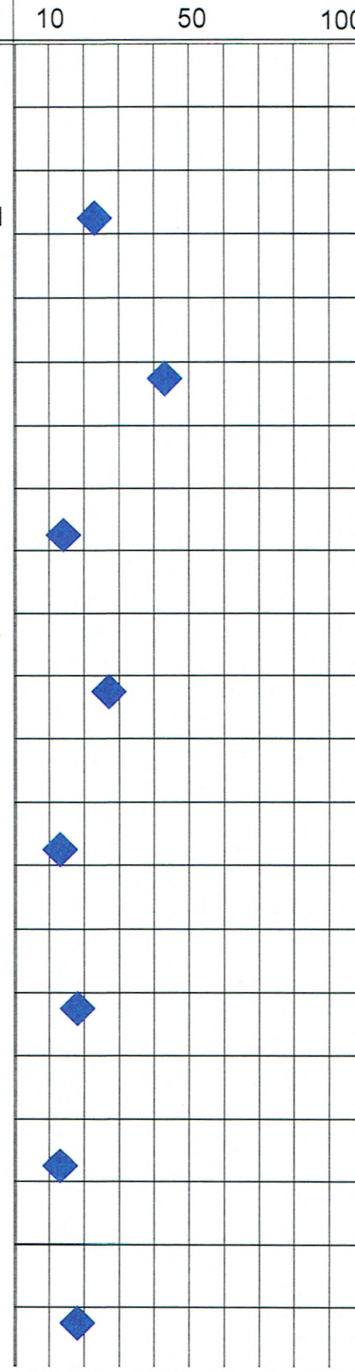
Project :	**Emrick Property**		
Project No.	1455	**Date :**	2-27-17
Client :	Burnstead	**Elevation**	160 feet
Location:	Upper Plat Area	**Logged By:**	Paul Bonifaci

Boring No. B-2

Depth (ft)	Soil Description	Water Level	USCS	Moisture Content (Laboratory Results)
0 to -14	Silty Sand: Tan to grey, partially to cemented layers of pebbly silty Sand to pebbly sandy Silt (Till-like), moist , medium dense		SM ML	
-6				14.3%
-11				9.9%
-14 to -42	Silt: Blue grey, clayey sandy Silt with some pebbles, stiff to very stiff, moist to damp low to moderate plasticity, non-bedded, non-jointed, massive	▼ (approx. -15)	ML	
-16				11.8%
-21				10.8% qu=3.0 tsf
-26				19.0% qu=2.5 tsf
-31				11.2% qu=2.5 tsf
-36				11.4% qu=2.0 tsf
-41				13.7%

SUBSURFACE PROFILE — SAMPLE — △ Standard Penetration Resistance (SPT) — Blows/Foot



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Date : March 2017	Project Name: Emrick Property	Figure 12

Project :	**Emrick Property**		
Project No.	1455	**Date :**	2-27-17
Client :	Burnstead	**Elevation**	160 feet
Location:	Upper Plat Area	**Logged By:**	Paul Bonifaci

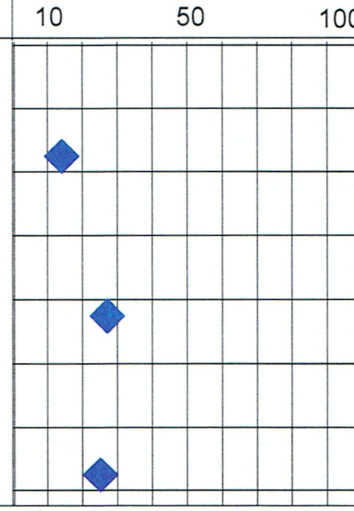
Boring No. B-2

Depth (ft)	Soil Description	Laboratory Results / Moisture Content
-42		qu=2.5 tsf
-43	Blue grey, clayey sandy Silt with some pebbles, stiff to very stiff, moist to damp low to moderate plasticity, non-bedded, non-jointed, massive	
-44		
-45		
-46		12.5%
-47		qu=1.75 tsf
-48		
-49		
-50		
-51	Groundwater at 15 feet	13.6%
-52	Boring terminated at 56.5 feet	
-53		
-54		
-55		
-56		

SUBSURFACE PROFILE — Depth (ft), Soil Lithology, Soil Description; SAMPLE — Water Level, Sample, USCS

△ Standard Penetration Resistance (SPT)

Blows/Foot



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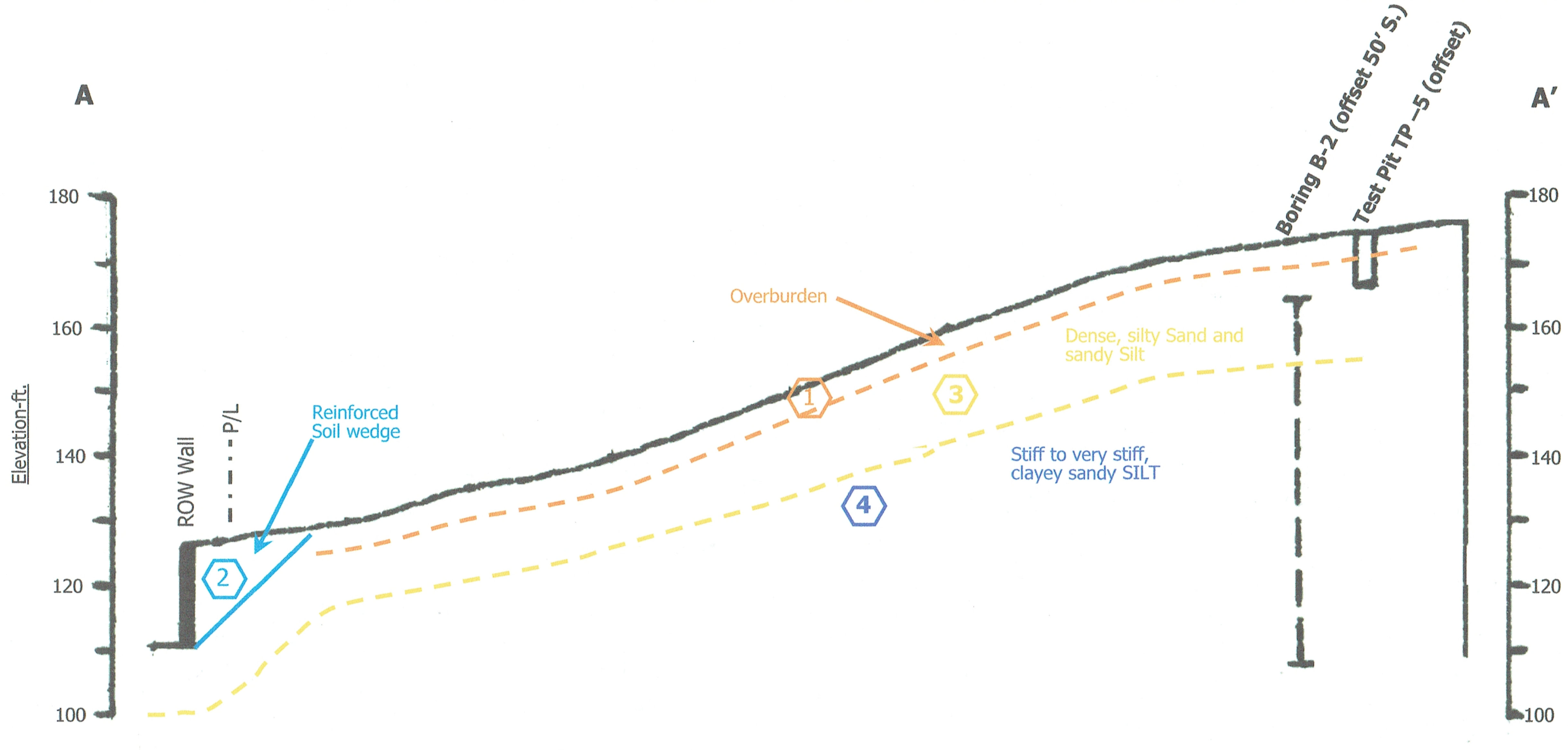
Date : March 2017	Project Name: Emrick Property	Figure 12



Soil Parameters Used in Slope Stability Analysis

1 **Soil Layer 1** cohesion c=200 psf, friction angle = 34 degrees

2 **Soil Layer 2** cohesion c=2500 psf, friction angle = 40 degrees

3 **Soil Layer 3** cohesion c=1500 psf, friction angle = 38 degrees

4 **Soil Layer 4** cohesion c=1000 psf, friction angle = 34 degrees

Approx. Scale (H:V)

0 50′

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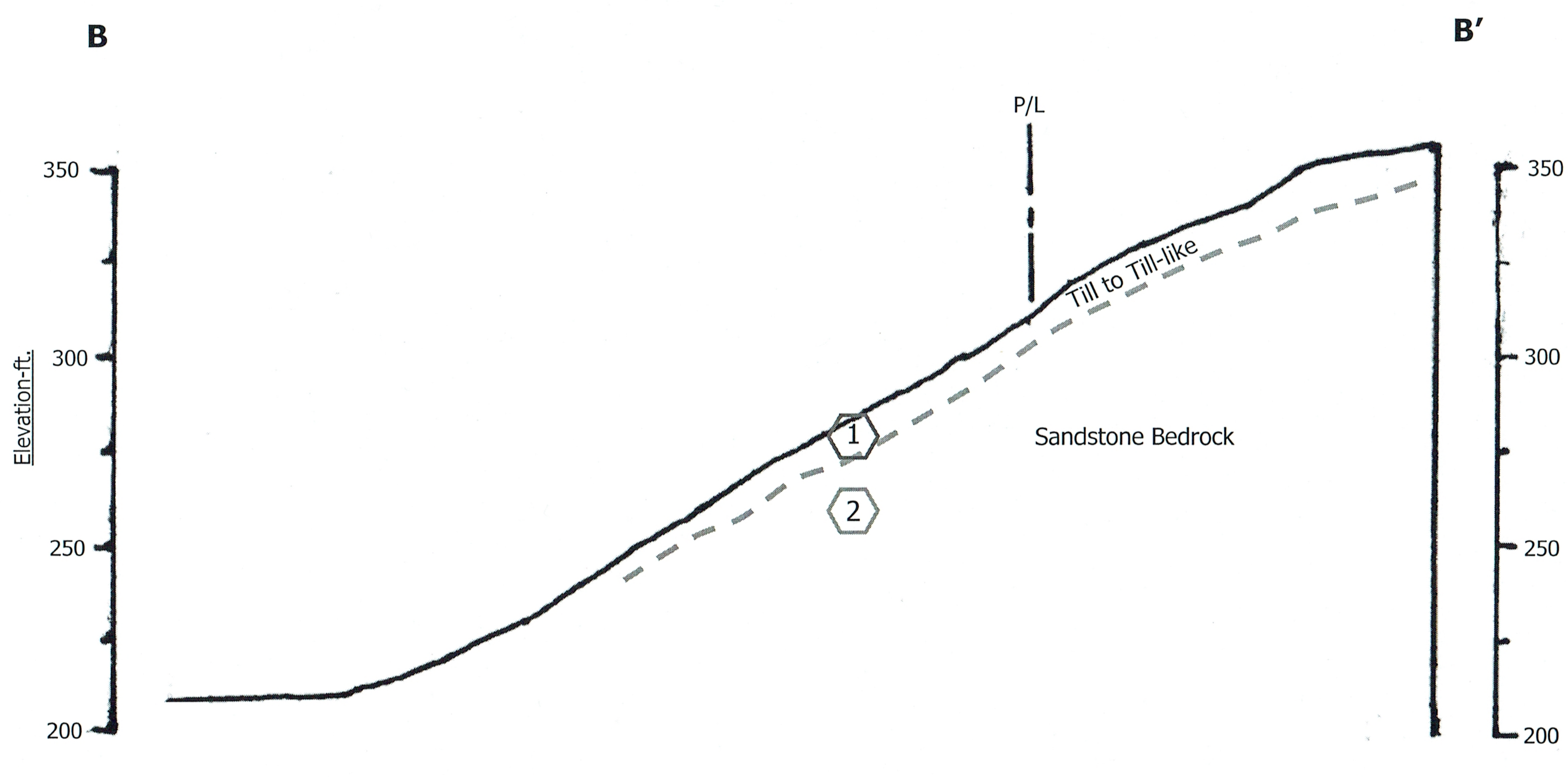
Kirkland, Wash.

Cross Section A—A′
Emrick Property
Issaquah, Washington

Proj. No. 1455	Date : March 2017	Figure 13



Soil Parameters Used in Slope Stability Analysis

1 Soil Layer 1 cohesion c=200psf, friction angle = 33 degrees

2 Soil Layer 2 cohesion c=1500 psf, friction angle = 38 degrees

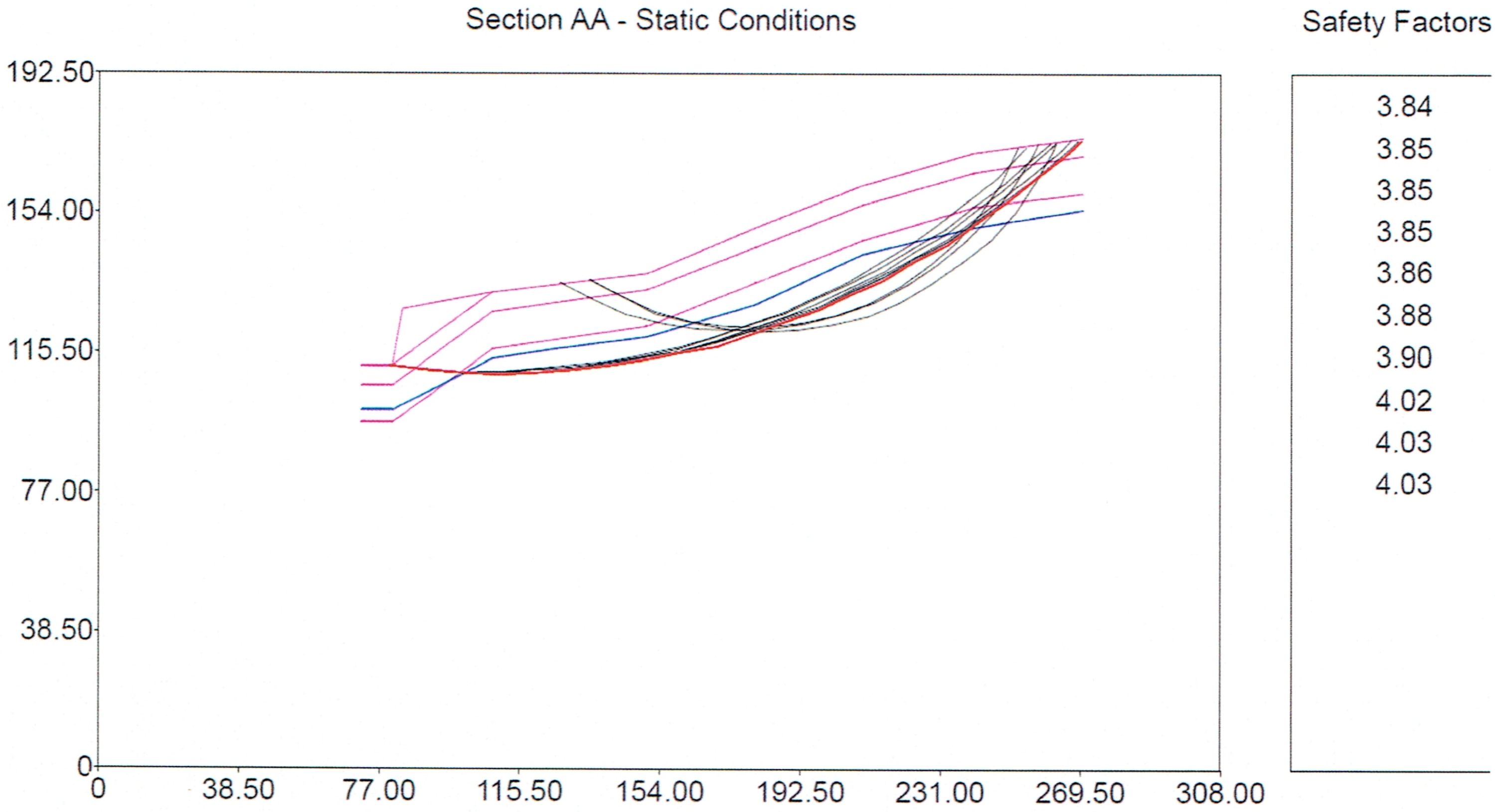
Approx. Scale (Horiz. and Vert.)

0 100′

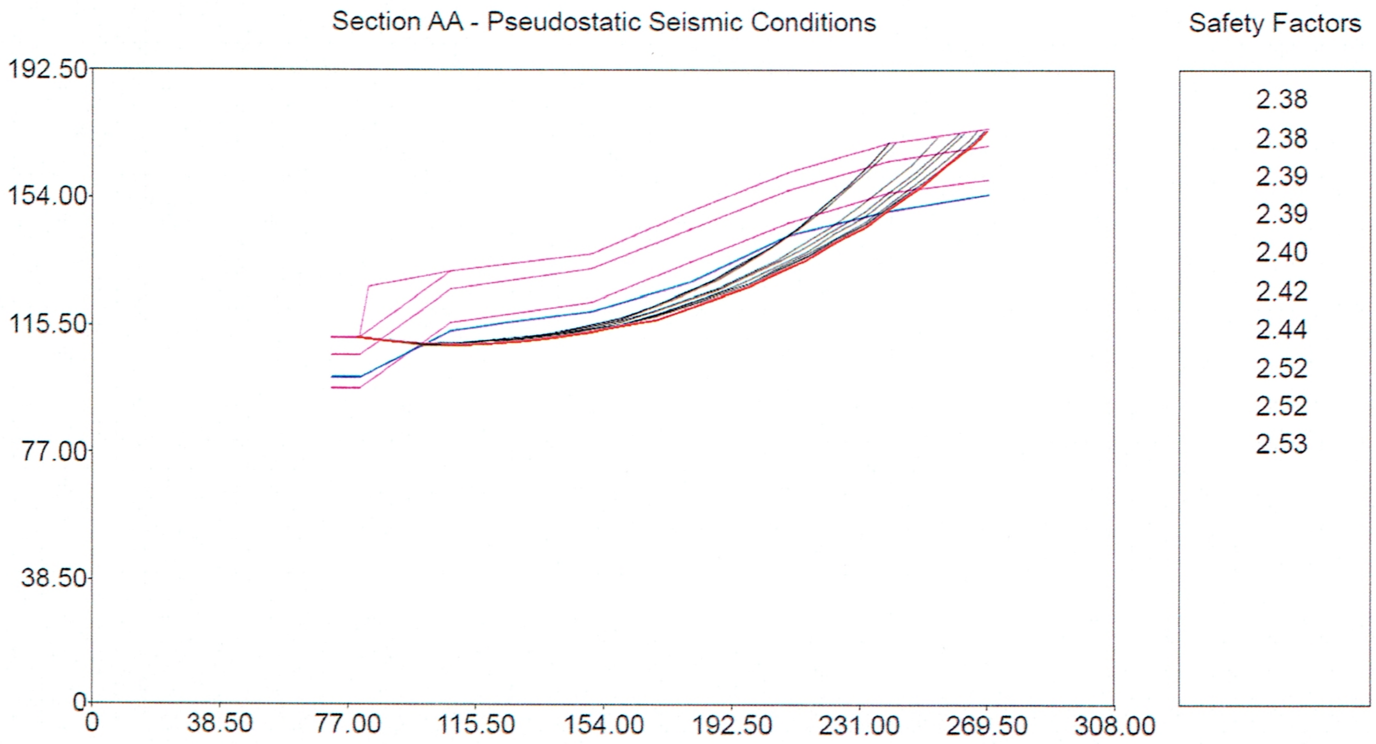
ABPB Consulting Geotechnical Consultants Kirkland, Wash.		
Cross Section B—B′ Emrick Property Issaquah, Washington		
Proj. No. 1455	Date : March 2017	Figure 14

Appendix A
Slope Stability Analysis Results

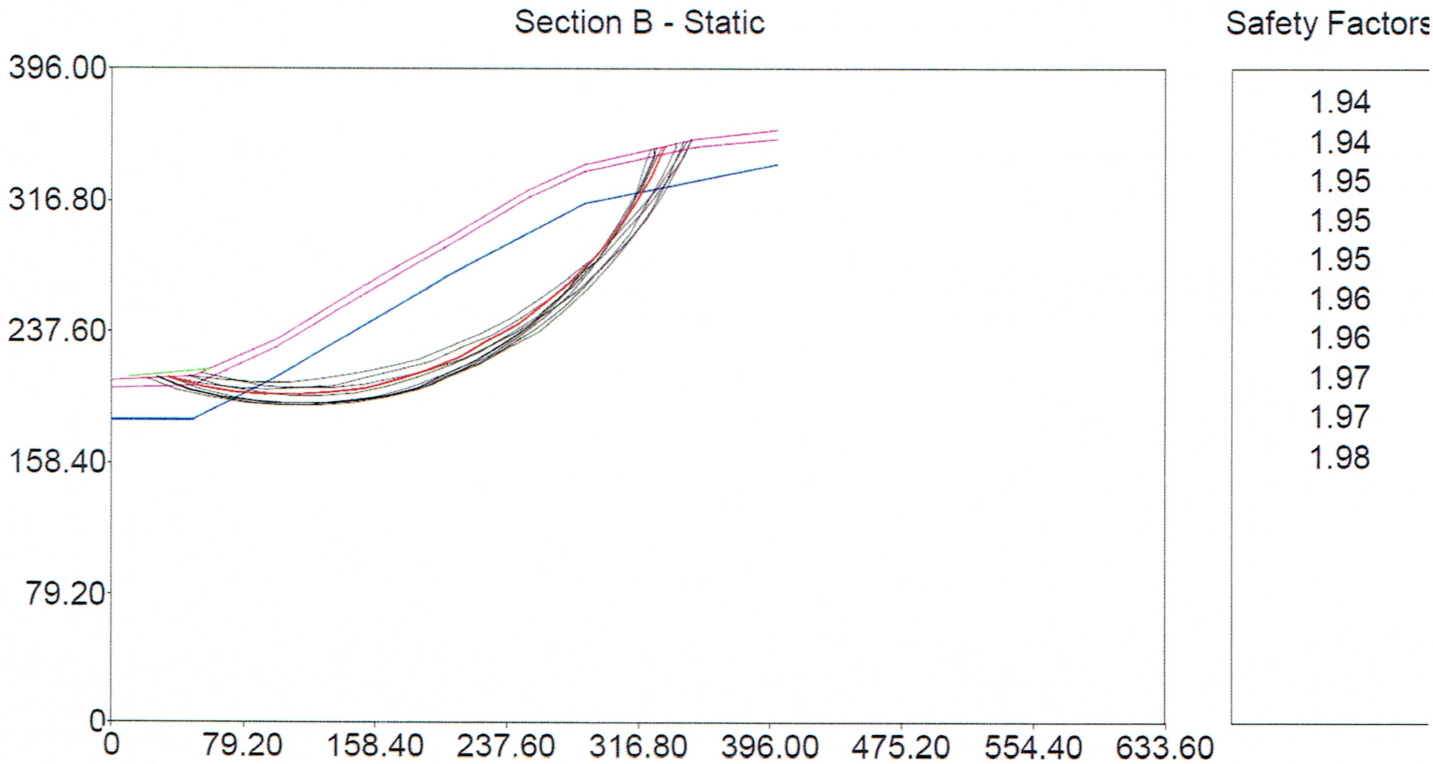
Emrick Property



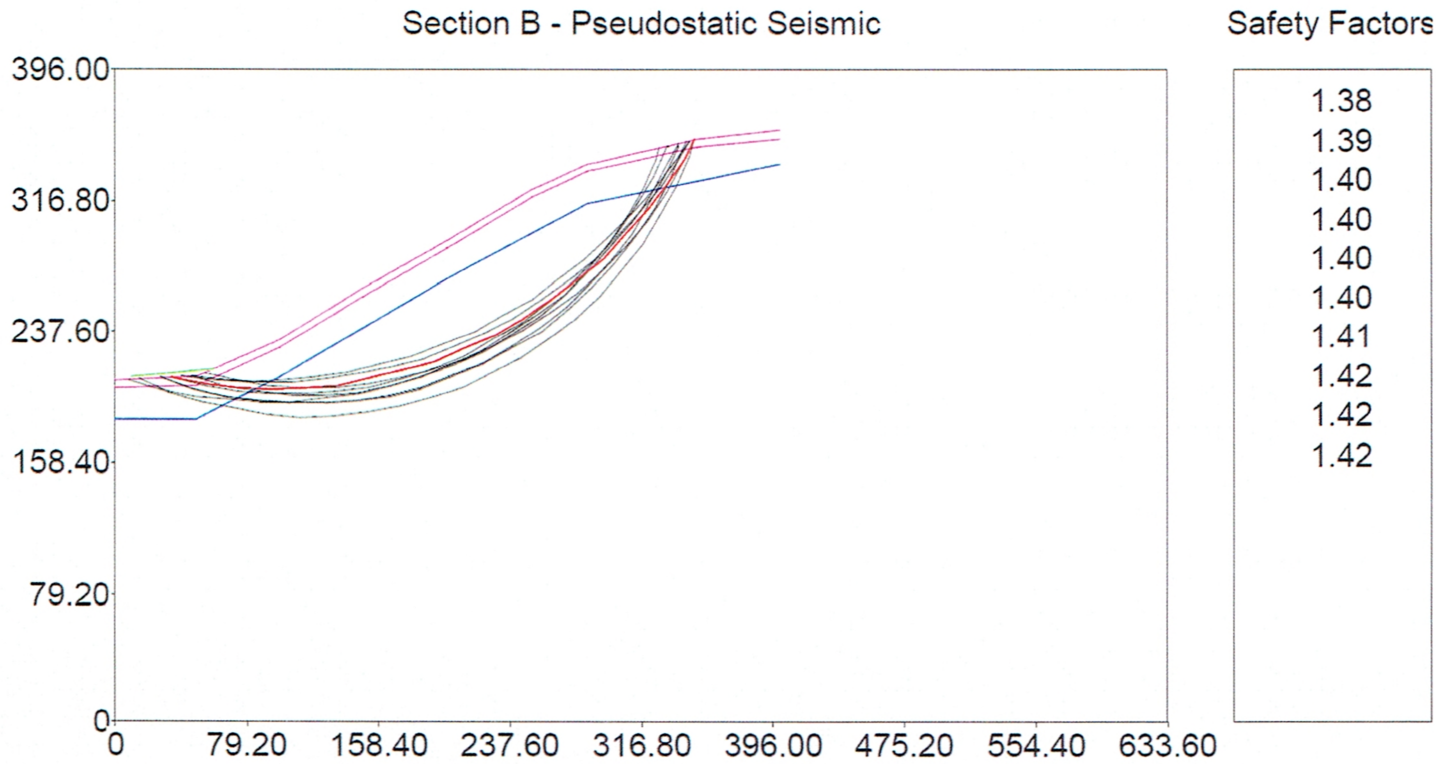
Emrick Property
Slope Stability Analysis
Static Conditions
Section A – A'
Project 1455 March 2017



Emrick Property
Slope Stability Analysis
Pseudostatic Seismic
Conditions
Section A – A′
Project 1455 March 2017



Emrick Property
Slope Stability Analysis
Static Conditions
Section B -B′
Project 1455 March 2017



Emrick Property
Slope Stability Analysis
Pseudostatic Seismic
Conditions
Section B -B′
Project 1455 March 2017